

1931

First progress report on column tests at Lehigh University, Proc. AC1, Vol. 27, (1931), p. 677 (31-1)

W.A. Slater

I. Lyse

Follow this and additional works at: <http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports>

Recommended Citation

Slater, W. A. and Lyse, I., "First progress report on column tests at Lehigh University, Proc. AC1, Vol. 27, (1931), p. 677 (31-1)" (1931). *Fritz Laboratory Reports*. Paper 1125.
<http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports/1125>

This Technical Report is brought to you for free and open access by the Civil and Environmental Engineering at Lehigh Preserve. It has been accepted for inclusion in Fritz Laboratory Reports by an authorized administrator of Lehigh Preserve. For more information, please contact preserve@lehigh.edu.

Corrected Office
Copy. 1/3/31
15243
1954

REINFORCED CONCRETE COLUMN INVESTIGATION;
FIRST PROGRESS REPORT ON TESTS

MADE AT
LEHIGH UNIVERSITY

by

W.A. Slater** and Inge Lyse*

I - INTRODUCTION

The tentative program and the results of the preliminary tests of the concrete for the reinforced concrete column investigation being carried out under the sponsorship of the American Concrete Institute was published in the JOURNAL for April, 1930. The parts of the report presented herein have been prepared by those having charge of the tests at Lehigh University. The authors accept the responsibility for the statements made. An effort has been made to avoid all generalizations but to present the data as fully as possible at the present time and to point out the important relations indicated in Series 1 and 2. ~~Committee 105 has not been able, in the time available, to prepare its report.~~

** Director, Fritz Engineering Laboratory, Lehigh University.
Chairman, Committee 105

* Assistant Engineer, Portland Cement Association,
In immediate charge of column
tests at Lehigh University

FRITZ ENGINEERING LABORATORY
LEHIGH UNIVERSITY
BETHLEHEM, PENNSYLVANIA

II - Series 1. Study of End Conditions

1. Purpose and Program - The purpose of Series 1 of this investigation was to study the effect of different end conditions on the strength and other properties of the columns, and from the results obtained decide upon the end condition to be used in the remaining series. This series was therefore completed before the others were begun.

The columns included in this series had a diameter of 8 in., an overall length, or shaft length, of 60 in., and were tested at an age of 28 days. All columns contained eight 1/2-in. square longitudinal reinforcing bars and a 3/16-in. wire spiral of 1.35-in. pitch. This corresponds to about 4 percent of longitudinal, and 1.2 percent of lateral reinforcement. The concrete was designed for a strength of 3500 lb. per sq.in. at 28 days as determined on 6 by 12-in. cylinders at the time of testing the columns.

Series 1 included three types of end condition. In the first type there were no capitals, and the ends of the longitudinal reinforcing bars were flush with the ends of the column. The second type had 14-in. diameter capitals. The third type had dowels and no capitals. Two groups of columns of the third type were tested. One group had 13-in. dowels which lapped over the reinforcing bars for a length of 20 diameters. The other group had 18-in. dowels providing for a

30-diameter lap. There was the same number of dowels as of the longitudinal bars, and both were of the same grade of steel. *Two columns were included in each group.*

2. The Concrete - The concrete used in this series was of a 1 : 2.4 : 3.6 mix by weight. The cement content was 5.4 sacks per cu.yd. of concrete. The nominal water content was 40.5 gal. per cu.yd. of concrete or 7.5 gal. per sack of cement. The concrete was mixed in a kettle-shaped mixer of about 2.5 cu.ft. capacity. The dry materials were mixed for one minute. Water was then added and the mixing was continued for three minutes more. Six 6 by 12-in. cylinders were made with each column, three for test at the age of 28 days, and three for test at 56 days.

Table 1 gives information on the properties of the concrete used in this series.

3. The Reinforcement - The longitudinal reinforcing bars were donated by the Bethlehem Steel Company of Bethlehem, Pennsylvania. These were plain bars having an average yield-point stress of 48,000 lb. per sq.in. and an average ultimate strength of 76,850 lb. per sq.in. The test results of five coupons taken from five different bars show a good degree of uniformity.

Coupon Number	1	2	3	4	5	Av.
Yield-Point Stress [*]	47,520	48,800	48,040	47,560	48,120	48,000
Ultimate Stress [*]	76,680	77,000	76,760	76,800	77,000	76,850

^{*}Stresses given in pounds per square inch

The lateral reinforcement was donated by the American Steel and Wire Company of Chicago, Illinois, and fabricated into spirals by the American System of Reinforcing. It consisted of 3/16-in. wire spirals having a pitch of 1.35 in. A definite yield point could not be detected for the spiral reinforcement. Therefore only ultimate strength was obtained.

Coupon Number	1	2	3	4	5	6	7	8	Av.
Strength lb. per sq. in. No. Test	89,340	83,570	85,730	83,930	83,570	87,170	77,450	84,390	

It will be noted that the lateral reinforcement was less uniform but about 10% higher in average strength than the longitudinal reinforcement.

4. Fabrication of Reinforcement - All the reinforcement for each column was assembled into a unit before being placed in the mold. Four of the longitudinal bars had six 1/2-in. steel cubes welded to the side so as to form a continuous row of five

10-in. gage lengths. The end cubes were placed five inches from the ends of the column or ends of the shaft.

The spirals had three spacers which were placed approximately 120° apart. Eight longitudinal bars were fastened with wires to the inside periphery of the spirals approximately 45° apart.

5. Making Columns - In Group 1 the longitudinal bars were flush with the ends of the column. This provided for transferring the load from the machine directly upon both the concrete and the steel. In the making of all columns, the concrete was filled to the top of the molds. A settlement of about $1/16$ -in. took place during the setting, due to the consolidation of the mass. Neat cement paste having a water-cement ratio of 0.3 by weight, re-mixed at four hours, was used as capping material. Planed steel plates used for the capping were worked down upon the ends of the longitudinal bars, bringing them flush with the surface of the concrete. The caps were usually about $1/16$ -in. thick.

The columns of Group 2 had a capital at each end. The shaft had the dimensions of the plain columns. The capitals were 18 in. in length and 14 in. in diameter. The longitudinal bars extended a distance of 30 diameters, or 15 in. into the capital. The spiral extended six inches into the capital. Wire hoops were placed in each capital

to prevent its falling apart in case of splitting during the testing. The form used for making the columns of Group 2 is shown in Fig. 1.

The columns of Group 3 had dowels of such a length as to form a 20-diameter splice. The longitudinal reinforcing bars extended to within three inches of each end; and the dowels were flush with the ends. Group 4 was of the same type as Group 3 except that a 30-diameter splice was used. Screws which passed through the end plates and into the ends of the dowels, held the dowels flush with the surface of the concrete and kept them in position during the placing. Fig. 2 shows the placing of the top plate on a column having dowels. The mold was filled to the top and the dowels were then forced into the concrete until the plate rested on the top of the mold. At the time of capping, the screws were removed and the end plate lifted. The capping was done in the ordinary manner.

In making the columns, the concrete was placed continuously and was rodded into place with 1/2-in. steel rods. The mold was tapped with a hammer in order to consolidate the mass. At the age of 24 hours the molds were removed, and a 1/4-in. diameter by 1/2-in. steel plug was set in the concrete with neat cement paste beside each lug on the longitudinal bar. The columns were then given three coats of

linseed oil and placed in the moist room. One day before testing, each column was removed from the moist room and gage holes drilled in all the plugs and lugs. Five continuous 10-in. gage lengths were thus obtained on both the steel and the concrete on elements 90° apart, around the column. Elements 1 and 3 were at opposite ends of one diameter, and 2 and 4 at opposite ends of the other.

6. Testing - The columns were cured in the moist room until they reached an age of 28 days, and then were tested to failure in an 800,000-lb. vertical screw testing machine. For the columns without capitals, an eight-inch spherical bearing block was placed between the head of the machine and the top of the column. The columns with capitals required a larger spherical bearing block and a 17-in. block was used. It was so heavy that it was placed at the bottom instead of the top. The column was plumbed by means of plumb bobs.

Before each column with plain ends was loaded, a complete set of strain-gage readings was taken. For the columns with capitals, a load of 200 lb. was applied before the initial readings were taken. With the spherical bearing block at the base of the column, this load was necessary to hold the column in line.

Five 10-in. gage lengths were observed on four rows of lugs on the steel bars with corresponding readings on the plugs in the concrete. This makes a total of 20 gage lengths on the steel, and 20 in the concrete. Each gage length was observed twice, so that 80 observations were made for each set of readings.

The testing crew consisted of two observers and a recorder. The observers took readings on opposite ends of the same diameter simultaneously. The order used in taking the measurements was as follows: Readings were taken from top to bottom on the steel in rows 1 and 3, and then from bottom to top on the concrete. The same order was used for readings on rows 2 and 4. A second set of readings was taken in a similar manner. Ordinarily the maximum variation between the readings of the two sets was 0.0002 in. as read on the dial of the Whittemore strain gage used. If a larger variation occurred a check reading was taken.

The load was applied in increments of approximately 25,000 lb. A complete set of observations was made for each load. The beam of the machine was rebalanced after the observations so that the load, both at beginning and end of observations, was recorded. This procedure was followed until the deformations went beyond the range of the strain gage on some of the gage lengths.

The time required for each loading, that is, from one application of the load to the next, varied. After some experience, the time for each set of observations ranged from seven to eight minutes.

7. Strength of Columns - All columns had 4 percent of longitudinal reinforcement and 1.2 percent of spiral reinforcement. The concrete was of the same mix and water-cement ratio throughout the series. Due to minor variations, however, slight differences in strengths were obtained. The test results of the control cylinders are given in section 2. The major difference in strength of the columns is, therefore, attributed to the end condition. Table 2 gives the maximum strengths of the columns having different end conditions. Fig. 3 to 7 inclusive, show the types of failure for the groups. The columns of Group 1, shown in Fig. 3, failed near the center. The columns with capitals, shown in Fig. 4, failed about ten inches below the capital. The columns with dowels failed at the ends of the dowels, either at the top or bottom of the column, as illustrated in Fig. 5 and 6. Fig. 7 shows the details of the failure of columns having dowels. After removal of the load a gap was generally found between the end of the reinforcing bar or dowel and the concrete. This is illustrated in Fig. 7. At the ends of the columns also the bars had been pulled back into the concrete slightly. This was

true in the columns with continuous bars as well as in those with dowels. It appears that for some reason the concrete elongated more than did the bars on removal of the load. The cause of this phenomenon has not been explained, but it may have been due partly to the buckling of the bars as shown in Fig. 3 and partly to the assistance of the spiral reinforcement in maintaining the integrity of the concrete.

In order to make a comparative study of the effect of the end condition on the strength of the columns, the variation in the strength of the concrete must be taken into account.

It is shown in Series 3 that the load carried by the plain columns was very nearly 85 percent of the strength of the control cylinders regardless of the strength of the concrete. Assuming that the concrete in the core of a spirally reinforced column will carry the same amount of load as in a plain column, the remaining load may be attributed to the presence of the steel, both spiral and longitudinal. In computing the load carried by the concrete the area used is the core area specified by the American Concrete Institute Building Code, that is, "the area within the outer circumference of the spiral hooping." As the outer diameter of the spirals was 7.75 in. and as the sectional area of the longitudinal bars used was two sq. in., the net core area of the concrete was:

$$\frac{\pi}{4}(7.75)^2 - 2 = 45.1 \text{ sq.in.}$$

The load attributed to the concrete was then $0.85 \times 45.1 f'_c$, where f'_c = unit strength of the control cylinders. Table 3 gives the computed division of load between concrete and reinforcement at the time of ^{failure}~~failure~~. The values given in this table have been plotted in Fig. 8. The upper line in this figure represents the proportionate total loads, and the lower line represents the proportionate loads attributed

to the reinforcement in the different groups. The fact that the curve for load carried by the steel is a straight line is due to a suitable selection of horizontal scale. Fig. 8 indicates that in order to obtain the same effect due to the reinforcement in a column with spliced ends, as in a column with bars flush with the ends, the length of the splice would have to be 50 times the diameter of the bars. The capitals apparently gave the equivalent of a splice of 50 diameters.

The load carried by the steel would probably have been no different if the cylinder strength had been exactly 3500 lb. per sq.in. for all groups. Assuming that this is true the strength of the reinforced column for 3500-lb. concrete has been estimated by adding to the load carried by the reinforcement 85 percent of the assumed cylinder strength of 3500 lb. per sq.in. The computations are shown in Table 4 and the column strengths corrected in this manner are shown in Fig. 8.

Fig. 8 indicates that the columns with the longitudinal steel flush with the ends of the column, developed higher strength than the columns with capitals or spliced reinforcement. The comparative ease of making this type of column makes it less expensive than the columns having dowels and much less expensive than the columns with capitals.

8 Deformations - In order to study the relation between deformations at different elevations, the average deformations for each group of columns have been plotted in Fig. 9 to ¹²~~11~~ inclusive.

Fig. 9 shows the average deformations for the columns of Group 1. This figure illustrates that at low loads the uniformity in the elastic properties at different elevations was good. For loads of 100,000 lb. or less, the deformations were practically equal at all elevations of the column. For higher loads, however, the deformations at the center elevation of the column were considerably larger than those at the ends. The most satisfactory explanation of this phenomenon seems to be the possibility of lack of homogeneity of the concrete. At low loads the steel and the concrete deformed together. At higher loads this equality of deformations continued near the ends, but near the center the deformations in the steel exceeded those in the concrete. The excess of deformation in the steel over that in the concrete increased as the load increased and failure occurred at the center as shown in Fig. 3.

Fig. 10 shows the average deformations of the columns having capitals. It is seen from the figure that the deformation of the steel was nearly equal to that of the concrete for all loads. It is noted, also, that the deformations at different elevations were nearly equal at low load, while for high loads the deformation was much greater at the top than at the bottom.

Fig. 4 shows that the failure occurred near the top of the shaft as would be expected from the behavior described. Slightly weaker concrete just below the capital, may have caused the deformations near the top to be larger than those near the bottom. A noticeable eccentricity in the loading (see Fig. 18) probably added to the stresses at the top of the shaft.

Fig. 11 shows the deformations at different elevations for the columns of Group 3. These columns had dowels which provided for a 20-diameter splice. As before, the deformations were nearly equal at different elevations for low loads. In general, the steel and concrete deformed together until a load of about 60% of the maximum strength of the column was reached. At this load the concrete in the bottom section deformed much more than did the steel, indicating that slipping of the bars must have taken place. As the load increased, the slipping between steel and concrete became more pronounced, and near maximum load it was noted that the concrete had moved away from the lugs on the steel forming a perceptible gap. The concrete spalled at the ends of the dowels and at the ends of the longitudinal bars at practically the same load, indicating that dowels and bars began to slip simultaneously.

Fig. 12 shows the deformations at different elevations for the columns of Group 4. These columns had dowels which lapped over the longitudinal reinforcing bars for a distance of 30 diameters. The figure indicates that the concrete and the steel deformed together throughout the test, for points at the center and below, and also for points above the center, until about 75 percent of the maximum load had been reached. At this load a slip occurred between steel and concrete near the top of the column, and the deformations of the concrete there became much larger than those of the steel. The concrete deformed very fast while the stress in the steel did not increase materially. The spalling of the columns occurred at the ends of the longitudinal bars and at the ends of the dowels as in Group 3. The failure of these columns was very similar to that of Group 3, except that the columns of Group 4 failed at the top while those of Group 3 failed at the bottom.

For both groups of columns having dowels, the deformations at all loads were smaller near the ends than at the center. This probably was due to the fact that on account of the lapping of the dowels the percentage of steel at the end section was twice the percentage at the center section.

9 Load Distribution - In Fig. 13 to 16, inclusive, the average deformations are plotted for the different loads on the columns. In addition, the loads corresponding to the strains in the steel are represented by a broken straight line. This line is carried to a strain of 1600 millionths, corresponding to a yield point stress of 48,000 lb. per sq. in. within which the modulus of elasticity of the steel was assumed as 30,000,000 lb. per sq.in. No determination of the modulus of elasticity of steel was made in this series of tests. The assumed division of the maximum load carried by the columns is shown on the right-hand side of the diagrams.

Fig. 13 shows the deformation diagram for the columns of Group 1. The assumption has been made that the concrete, the longitudinal steel, and the spiral have jointly contributed to the column strength. The load attributed to the concrete at the maximum load is the product of 55 percent of the average cylinder strength and the core area of the column, that is, $0.55 \times 3140 \times 45.1$, or 120,500 lb. The load attributed to the longitudinal steel is the product of the yield-point strength and the area of the steel, that is, $48,000 \times 2$, or 96,000 lb. The sum of the loads attributed to the concrete and to the longitudinal steel ($120,500 + 96,000$ or 216,500 lb.) has been subtracted from the column strength (258,900 lb.). This difference ($258,900 - 216,500 = 42,400$ lb.) has been attributed to the spiral.

According to the above assumption, the load attributed to one percent of longitudinal steel is $\frac{96000}{4}$, or 24,000 lb. The 1.2 percent of spiral apparently contributed 42,400 lb., indicating that one percent of spiral was nearly as effective as 1.47 percent of longitudinal reinforcement for the columns of Group 1.

The deformations at top and bottom gage lengths for the columns of Group 2 and the division of load, are shown in Fig. 14. The total load on the column has been distributed between the concrete, the longitudinal steel, and the spiral. It is noted that the load attributed to the 1.2 percent of spiral is 32,700 lb., or 1.14 times as much as for an equal percentage of longitudinal reinforcement.

Fig. 15 shows the deformation diagram and the division of the load for the columns of Group 3. Here the sum of the yield-point strength of the steel and the load attributed to the concrete is almost equal to the maximum strength of the column. The spiral did not seem to add anything to the strength of the columns, probably because failure was precipitated by the slipping of the dowels before the spirals could come into action.

The deformation diagram and the division of the load for the columns of Group 4 are shown in Fig. 16. In this case the strength of the column was about 11,000 lb. greater than the sum

of the yield-point strength of the steel and the load attributed to the concrete. This 11,000 lb. may be attributed to the spiral, but the spiral probably was not fully effective because of slipping of the dowels. It is of interest, however, to point out the fact that the 30-diameter lap was more effective than the 20-diameter lap.

10 Eccentricity of Loading - If a plane section remains plane during the loading, the sum of the strains ϵ_1 and ϵ_3 measured at the ends of one diameter, should be equal to the sum of the strains ϵ_2 and ϵ_4 at the ends of another diameter. This relation was found to hold approximately true for all the columns. The strain volume is, therefore, a cylinder with ends parallel in the case of centric loading, and non-parallel for eccentric loading. Assuming that the stress is proportional to the strain, this relation may be used to determine the size and shape of the stress cylinder and the distance of the center of pressure from the center of the section, that is, the eccentricity of loading. The center of pressure must pass through the center of gravity of the stress cylinder in order to establish equilibrium.

The following method was used in determining the position of the center of pressure. (see Fig. 17).

The volume of the stress cylinder is:

$$V = \pi r^2 \bar{\epsilon} E$$

where r = the radius of the cylinder

$\bar{\epsilon}$ = the average strain, that is,

$$1/4(\epsilon_1 + \epsilon_2 + \epsilon_3 + \epsilon_4) \text{ and}$$

E = modulus of elasticity

The bending strain, ϵ_b is: $\epsilon_b = r \tan \theta$, where θ

is the angle between the end planes of the stress cylinder

The bending moment is:

$$M = \frac{\epsilon_b}{c} EI$$

where I = moment of inertia of the cross-section

and c = distance from the neutral axis to the point at which ϵ_b is measured

Now

$$I = \frac{\pi r^4}{4} \quad \text{and } c = r$$

Then

$$M = \frac{r \tan \theta}{r} E \frac{\pi r^4}{4} = \frac{\pi r^4}{4} E \tan \theta$$

The distance e from the neutral axis to the center of gravity of the stress cylinder, that is, the eccentricity is:

$$e = \frac{M}{V} = \frac{\frac{\pi r^4}{4} E \tan \theta}{\pi r^2 \bar{\epsilon} E} = \frac{1}{4} \frac{r^2}{\bar{\epsilon}} \tan \theta$$

From the illustration:

$$\tan \theta = \frac{\epsilon_4 - \epsilon_2}{2r \cos \phi} = \frac{\epsilon_1 - \epsilon_3}{2r \sin \phi}$$

where ϕ is the angle between a plane passing through elements 2 and 4 and a vertical plane passing through the line of maximum and minimum stresses. The center of gravity of the stress cylinder will lie in the plane passing through the maximum and minimum strains at a distance, e , from the centre line of the cylinder.

The angle ϕ is determined from:

$$\frac{\epsilon_4 - \epsilon_2}{2r \cos \phi} = \frac{\epsilon_1 - \epsilon_3}{2r \sin \phi}$$

$$\text{or } \frac{2r \sin \phi}{2r \cos \phi} = \frac{\epsilon_1 - \epsilon_3}{\epsilon_4 - \epsilon_2} = \tan \phi$$

$$e = \frac{1}{4} \frac{r^2}{\Sigma} \tan \phi = \frac{1}{4} \frac{r^2}{\Sigma} \frac{\epsilon_4 - \epsilon_2}{2r \cos \phi} = \frac{r}{8\Sigma} \frac{\epsilon_4 - \epsilon_2}{\cos \phi}$$

Therefore the eccentricity is given by the formula:

$$\tan \phi = \frac{\epsilon_1 - \epsilon_3}{\epsilon_4 - \epsilon_2} \quad \text{and} \quad e = \frac{r}{8\Sigma} \frac{\epsilon_4 - \epsilon_2}{\cos \phi}$$

Thus the center of pressure may be easily determined from the readings obtained on the four elements 1, 2, 3, and 4. In Fig. 18 the center of pressure is given for top, center, and bottom elevations for loads of 25,000 and 200,000 lb. respectively.

This figure shows that the eccentricity was in all cases considerably less for the load of 200,000 lb. than for 25,000 lb. Also, it was generally less for the center and the bottom than for the top of the column. It may be shown that the ratio of bending stress to total stress is equal to the ratio $\frac{4e}{r}$, that is, four times the ratio of the eccentricity ^{to} of the radius of the column. For the columns without capitals this ratio, $\frac{4e}{r}$, was always less than 1/2. For the 200,000-lb. load it was generally much less. For the columns with capitals the ratio, $\frac{4e}{r}$, was greater than 1/2 in all cases for the load of 25,000 lb. and at the top it exceeded 1, that is, the bending stress in this case was greater than the average stress, $\frac{P}{A}$. However, for the load of 200,000 lb. the bending stress had been reduced to about one-half the average stress. For all the other columns at the load of 200,000 lb. the bending stress was generally less than 20 percent of the average stress $\frac{P}{A}$. The foregoing analysis has been based upon an assumed constant modulus of elasticity. The fact that in general the modulus is not constant will operate to reduce somewhat the eccentricity and the ratio of bending to average stress.

// Summary - The columns having longitudinal reinforcing bars flush with the concrete at the end of the column (Group 1) appeared to develop the full strength of ^{their} ~~its~~ component parts and failed near the center without showing local weakness.

The columns with capitals developed high bending stresses and failed near the top.

The columns with dowels showed less strength than did those with continuous bars. Failure occurred at the ends of the bars and at the ends of the dowels.

The superiority of the columns with continuous bars flush with the surface of the concrete and having no capitals, together with the fact that they were the less expensive to make and test, resulted in the adoption of this type of column for the remainder of the investigation.

III - SERIES 2. EFFECT OF HOLDING THE LOAD
FOR FOUR HOURS

12. Purpose and Program - The purpose of this series of tests was to ascertain whether the holding of load for a short period such as four hours, would have any effect on the strain and the ultimate strength of the column. There was the added purpose of comparing strengths of columns with three different grades of longitudinal reinforcement, structural, intermediate, and rail steel grades. Variations in percentage of reinforcement, and strength of concrete also were introduced as is indicated in Table 5.

The columns had an outside diameter of 8-1/4 in., a core diameter of 8 in., and an overall length of 60 in. The longitudinal bars were 60 in. long and the ends were milled, thus insuring direct bearing of both steel and concrete on the end plates. *Three like columns were made for each group of plain concrete columns and two like for each group of reinforced concrete columns.*

13. Proportioning and Strength of Concrete* - The concrete strength was determined on moist-cured 6 by 12-in. cylinders at an age of 56 days. The concrete designed for

* The cement and aggregates used in this series came from the same supplies as those used in the Preliminary Investigation reported in the JOURNAL for April, 1930

a cylinder strength of 2000 lb. per sq.in. was of a 1 : 3.42 : 5.13 mix by weight of the dry materials. The assumption was made that the amount of water absorbed by the dry aggregates was 0.7 percent of their weight. The cement content was 4.0 sacks per cu.yd. and the net water content was 39 gal. per cu.yd. of concrete or 9-3/4 gal. per sack of cement. The test results showed an average cylinder strength of 2280 lb. per sq.in. and an average slump of 2-1/2 in. for all specimens included in this mix.

The concrete designed for a cylinder strength of 3500 lb. per sq.in. was of a 1 : 2.65 : 4.00 mix. The cement content was 5.02 sacks per cu.yd. and the net water content 39 gal. per cu.yd. of concrete or 7-3/4 gal. per sack of cement. The average strength was 3080 lb. per sq. in. and the average slump 3-3/4 in.

The concrete designed for a cylinder strength of 5000 lb. per sq.in. was of a 1 : 1.98 : 2.96 mix. The cement content was 6.5 sacks per cu.yd. and the net water content 39 gal. per cu.yd. or 6 gal. per sack of cement. The average strength was 5360 lb. per sq.in., and the slump was 5-1/2 in. It will be noted that the water content was 39 gal. per cu.yd. of concrete for all these mixes. A place-able concrete was produced with this water content and only a small variation was noted in the consistency as measured by the slump.

In addition, 11 columns with concrete designed to have a compressive strength of 8000 lb. per sq.in. were made. The mix used for this concrete was 1 : 1/2 : 1 by weight. The cement content was 15.3 sacks per cu.yd. and the water content was 46 gal. per cu.yd., or 3 gal. per sack of cement. This mix was so dry that it could not be properly placed by ordinary methods. It was therefore worked into place by means of an electric vibrator furnished by the Electric Tamper and Equipment Company. The amount of vibration required was quite extreme, and even then small pockets were found in the interior of the cylinders when they were broken. No such pockets were found in the plain columns but no examination was made of the interior of the reinforced columns.

Fig. 19 shows the relation between the strength of the concrete and the water-cement ratio used in the mix. The water-cement strength relation for the plastic mixes was a smooth curve very similar to ordinary water-cement strength curves. The relation between cement content and strength is given by the straight, broken line. In other words, as long as the water content per cubic yard of concrete was maintained constant with nearly constant consistency of the different mixes, the strength was directly proportional to the cement content.

The strength of the concrete designed for 8000 lb. per sq.in. proved to be only 7000 lb. per sq.in., and it falls below the water-cement strength curve defined by the other mixes. Neither does this strength fall upon the straight line determined by the other mixes when their strengths are plotted against the cement content. This 7000-lb. concrete contained 13.5 sacks of cement per cu.yd. and should have a strength of approximately 12,000 lb. per sq.in. in order to fall upon either of the strength curves shown in Fig. 19.

14. Modulus of Elasticity of Concrete - The modulus of elasticity was determined on one control cylinder from each column. The apparatus used consisted of two steel collars, 10 in. apart vertically, attached directly to the cylinders by means of two set-screws 180° apart. Opposite each of the set-screws a 1/10,000-in. Ames gage with its plunger pointing downward was clamped to the upper collar. A steel rod clamped to the lower collar opposite each set-screw extended upward and bore against the plunger of the Ames gage so that a direct reading of the total deformation in 10 in. was obtained. The results thus obtained were averaged for each type of concrete used and these averages are plotted in Fig. 20. The tangent modulus at a stress of 500 lb. per sq. in. was decided upon as the most reasonable value of the

modulus of elasticity. In Fig. 21 the moduli of elasticity have been plotted against the strength. For all except the vibrated concrete, the increase in the modulus was very nearly proportional to the increase in strength. Even the value for the vibrated concrete falls not far above the projected straight line determined by the other values.

The results obtained indicate that the method used for determination of the modulus of elasticity was satisfactory. The simplicity of this method makes it very convenient for determination of modulus of elasticity where greater accuracy is not possible on account of time and expense involved.

15. Reinforcement - The longitudinal Reinforcement consisted of intermediate, structural, and rail grades of steel bars. The intermediate and structural grades of bars were donated by the Concrete Steel Company of New York, and the rail steel by the Rail Steel Bar Association, Chicago. The bars were 22 ft. long when received. At the laboratory they were cut into four 5-ft. lengths and a 2-ft. test coupon. The four 5-ft. lengths were used in the same column. Four 1/2-in. diameter bars correspond to 1.5 percent of longitudinal reinforcement, eight 1/2-in. square bars correspond to 4 percent, and four 5/8-in. diameter plus four 3/4-in. diameter bars correspond to 6 percent of reinforcement. The coupons from each set of four bars were tested for yield-point

stress and ultimate strength. The summary of these tests is given in Table 6. Both yield-point stress and the ultimate strength of the coupons (based upon nominal areas) varied considerably. The variation in weights of bars of the same size indicates that the variation in strength was probably due to the variation of the cross-section from the nominal size of the bars, rather than differences in quality of the steel.

The yield point and ultimate strength for the structural grade were only slightly lower than those for the intermediate grade steel. The rail steel showed yield point and ultimate strength considerably higher than those for the intermediate grade. The marked influence of this difference will be pointed out later.

The lateral reinforcement consisted of wire spirals. The spiral wire was furnished by the American Steel and Wire Company, Chicago, and fabricated into spirals by the American System of Reinforcing, Chicago. The spirals had an outside diameter of 8 in., and an overall length of 59 in. Three extra turns were provided at each end of the spiral. A No. 5 wire having a 1.55-in. pitch gave 1.2 percent lateral reinforcement. A test coupon was attached to each spiral. The average tensile strength on these coupons is given in Table 6.

It will be noted that the ultimate strength of the No.5 wire was considerably above that of the intermediate grade longitudinal reinforcement.

The reinforcement for each column was assembled into one unit before being placed in the mold. For the purpose of carrying the strain gage holes, 1/2-in. steel cubes were welded to four of the longitudinal bars at intervals of 10 in. along their length, thus forming a continuous row of five 10-in. gage lengths on each bar. The end cubes were placed five inches from the ends of the column.

The spirals had three spacers which were placed approximately 120° apart. The longitudinal bars were fastened to the inside periphery of the spiral by means of wires.

16. Placing and Curing of Concrete - The mixing was done in a kettle-shaped mixer. The dry materials were mixed for two minutes, the water was then added, and the mixing was continued for four minutes more. One batch of concrete was sufficient for one column and its three control cylinders. For the plastic concrete continuous placing was used in the concreting of the columns. During the placing, the concrete was rodded continuously with a 1/2-in. steel rod and the mold was tapped with a hammer in order to consolidate the mass. The concrete to be placed by the vibrator was placed in ~~the~~ three equal layers. Each layer was vibrated for about 45 seconds.

About four hours after concreting, the columns were capped with a remixed neat cement paste. At the age of 24 hours the molds were removed, and a steel plug 1/4-in. in diameter and 1/8-in. long was set in the concrete opposite each lug on the longitudinal bars. Neat cement paste was used to embed them in the concrete. The plugs in the concrete and the lugs on the bars were coated with "Cosmic anti-rust preparation", and the columns were placed in the moist room. At the age of seven days the columns were removed from the moist room, holes were drilled in the plugs and lugs, strain gage observation were taken, and the columns were replaced in the moist room to remain until an age of 56 days was reached.

17. The Testing - At the age of 56 days the columns were removed from the moist room and placed in the 800,000-lb. screw-power testing machine. The testing ~~test~~ consisted of two men, an observer and a recorder. Strain gage readings were taken on the entire 40 gage ^{lengths} ~~lines~~ before applying any load and again at each increment of load. About 10 increments were used. The time required for a full set of observations was about seven minutes. With the "fast" loading the weighing beam was re-balanced after completion of each set of readings and a new increment of load was applied immediately.

With the "slow" loading the procedure was similar to that for fast loading up to about 70% of the maximum load. From that time on four hours elapsed between successive increments of load, and readings were taken immediately after completion of one increment and again just before starting of the next one. During the interval between increments the load was kept within about 2000 lb. of that intended. This was done by "bringing up the load" whenever it had fallen off appreciably. The frequency with which the machine had to be operated to maintain the load varied from a few minutes to more than half an hour within any given period between increments. Near the maximum load the frequency with which it was necessary to bring up the load was much greater than for the lower loads. To prevent drying out during the four-hour period in which the load was held, the column was enclosed in a canvas jacket which was kept moist by sprinkling. All the columns were photographed after completion of the test. A measure of the reliability of the experimental work of this investigation is found in the close agreement of measured strains (1) between the steel and the concrete of the same column, (2) between companion columns of any given group, (3) between two groups of the same kind, one of which was used for fast loading and the other for slow loading. This

agreement indicates not only good strain gage work but uniformity of making, capping, and curing of the test specimens.

18. Plain Columns - The ultimate strengths of the plain columns and of their control cylinders are given in Table 7, also the strengths of the columns ^{stated} tested as a percentage of the strength of the control cylinders. The average strength of the plain columns was approximately 65 percent of the strength of the cylinders and the uniformity of both the cylinders and the columns, was good. The plane of failure generally made an angle of about 30° with the axis of the column as illustrated in Fig. 22. Failure generally occurred within the upper half of the column, a fact which is consistent with the observation in Series 1, that the eccentricity of loading was largest near the top.

The columns of 8000-lb. concrete failed explosively. Those with 5000-lb. concrete failed less abruptly but almost without warning. The columns of 3500-lb. concrete failed gradually and in some cases it was possible to get them out of the machine whole. The columns with 2000-lb. concrete settled down gradually under the load and were removed from the machine in one piece

The stress-strain curves for the cylinders and the columns tested with fast loading are given in Fig. 23, 24, 25, and 26, for the four strengths of concrete. Each curve

is the average for all the specimens of its kind. The solid lines represent the columns and the dotted lines represent the cylinders. The outstanding features of these figures are the close agreement between the curves for the cylinders and those for the columns within the lower half, and the consistently greater deformation in the columns within the upper half of the curves. Generally the curve for the cylinders lies slightly above that for the columns even at low loads, indicating apparently, that there was some slight delay in getting the strain measuring apparatus used on the cylinders into action.

The modulus of elasticity given by the slope of the stress-strain curves for the columns for a stress of 500 lb. per sq.in. is in close agreement with that found for the cylinders.

Fig. 27 shows the stress-strain curves for fast and slow loading of columns designed for strengths of 2000, 3500, and 5000 lb. per sq.in. The horizontal portions of the curves represent the deformation which occurred during the four hours under practically constant load. The outstanding feature is the close agreement of these portions of the load strain curves in which the load was applied ^qrapidly.

The horizontal distance between the upper parts of the two curves for the same concrete strength represents the portion of the total deformation which is due to the holding of the load for four hours.

19. Relation Between Strains and Percentage of
Longitudinal Reinforcement - Fig. 28 and 29

show the stress-strain relations for columns of concrete designed for strengths of 8000 and 3500 lb. per sq.in. respectively. The spiral reinforcement amounted to 1.2 percent for all the columns, and the longitudinal reinforcement varied from 0 to 6 percent as indicated. The stress $\frac{P}{A}$ plotted in these diagrams is the load on the column divided by the total area within the outer circumference of the spiral, that is, by 50.3 sq.in., the sum of the sectional areas of the circumscribed concrete and the longitudinal reinforcement. The portion of this average stress that was considered to be carried by the longitudinal reinforcement is indicated by the broken straight lines in the lower part of the diagram. In constructing these lower lines the modulus of elasticity of the steel was taken at 30,000,000 lb. per sq. in. and the stresses were determined from the measured strains. The break in each line comes at the strain corresponding to the yield-point stress of the bars of the size used to secure the percentage of reinforcement indicated.

The stress-strain curves for the control cylinders made with the columns having only spiral reinforcement, have been plotted in ~~the same~~ Fig. 28 and 29. ^{9/50} In each figure the

curve for the cylinder is a dotted line and its agreement with the curve for the corresponding column is very good. Fig. 28 and 29 indicate much less departure of the strain in the spiraled columns from that in their control cylinders than is shown in Fig. 23 to 26 for the plain columns. The indication is that the presence of the spiral had a slight effect in reducing the longitudinal strain in the columns for stresses above about one-half the cylinder strength, but no effect for lower stresses.

For the concrete designed for a compressive strength of 6000 lb. per sq.in., Fig. 28, the vertical distances between the stress-strain curves for the columns with various percentages of longitudinal reinforcement are very nearly the same as the vertical distances between the corresponding straight lines representing the stresses carried by the reinforcement. That is, if the ordinates to the straight lines (the stresses carried by the longitudinal reinforcement) were subtracted from the ordinates to the stress-strain curves for the columns, the resulting curves would practically coincide with the curve for the column having no longitudinal reinforcement. This indicates that the presence of longitudinal reinforcement had no perceptible effect on the elastic behavior of the concrete designed for a strength of 6000 lb. per sq.in.

With the 3500-lb. concrete shown in Fig. 29 for fast loading and Fig. 30 for slow loading, the case was different. The vertical distances between the stress-strain curves for 0 and 1/2 percent reinforcement were considerably less than the ordinates of the line representing the stresses carried by 1-1/8 percent of reinforcement. With this exception, the conditions for the 3500-lb. concrete were about the same as those for the 3000-lb. concrete with the result that if the stresses carried by the reinforcement were subtracted from the total average stresses carried by the corresponding columns the resulting curves for the stresses carried by the concrete for the reinforced columns would practically coincide with each other, but would lie below the curve for the columns without longitudinal reinforcement. The behavior is as though the introduction of the longitudinal reinforcement reduced the effective sectional area of the concrete by a constant amount in excess of the area of the reinforcement used. The only explanation of the difference in behavior between the 3000 and 3500-lb. concrete which seems reasonable is based upon a possible difference in compactness due to the difference in the methods of placing. The 3000-lb. concrete was vibrated during the placing, and the 3500-lb. concrete was not.

20. Relation Between Strength of Column and

Strength of Concrete - Fig. 31 shows, as ordinates, the total loads carried by the columns and as abscissas, the strengths of their control cylinders. The lower curve represents the plain columns and the upper curves represent columns having 4 percent longitudinal and 1.3 percent spiral reinforcement. The spiral reinforcement was all of intermediate grade; the longitudinal included structural, intermediate, and rail grades of steel. The strengths of the columns increased quite consistently with the increase in strength of the control cylinders. The rate of increase in strength of the reinforced columns seems to have been nearly equal to that of the plain columns when net core areas* of concrete in the reinforced columns are considered. That is, the increase in the strength of the concrete was just as effective in a reinforced column as it was in a plain column. It is shown in a previous section that for a plain column the strength was 85 percent of its cylinder strength regardless of the strength of the concrete. Therefore the increase in strength of the reinforced columns would be equal to the net area of its concrete times 85 percent of the increase in strength of its control cylinders, regardless of the grade of longitudinal reinforcement.

* By the term "net core area" of the concrete is meant the area within the outer circumference of the spiral minus the sectional area of the longitudinal reinforcement.

In Fig. 32 the maximum loads carried by reinforced columns have been plotted as ordinates and the percentage of longitudinal reinforcement as abscissas. The percentage of spiral was 1.2 for all columns. The straight line representing the relation between the strength of the column and the percentage of reinforcement for a concrete strength of 3500 lb. per sq.in. is parallel to the line representing the same relation for the 3000-lb. concrete. ^{at} This is, the strength added to a reinforced column due to a given increase in the concrete strength was the same for all the percentages of longitudinal reinforcement used.

Fig. 33 shows the relation between the strength of the columns and the total yield-point strength of the longitudinal reinforcement. This figure is a combination of the previous two figures and shows that the increase in strength of columns due to increased strength of the concrete was practically independent of the yield-point strength of the reinforcement used in the columns. ¶ The effect of the strength of the concrete upon the deformation of columns having 4 percent longitudinal and 1½ percent spiral reinforcement is illustrated in Fig. 34. The strains for a given stress decreased quite regularly with the increase in strength of the concrete.

21. Relation Between Yield-Point Strength of Longitudinal Reinforcement and Strength of Column - The strength added by the longi-

tudinal reinforcement in the columns may be varied by changing either the percentage or the grade of reinforcing steel. The effect of various grades of steel upon the strength of columns of a given percentage of reinforcement and strength of concrete is illustrated in Fig. 31. Intermediate, structural, and rail grades were included in these tests, and the column strengths obtained with these grades of longitudinal reinforcement are shown in the three upper lines. The yield-point strength of the reinforcement is given in Table 6. Structural and intermediate grades had nearly equal yield-point stresses (40,840 and 43,560 lb. per sq. in.), but the rail grade had a much higher yield point stress (63,320 lb. per sq. in.). The lines representing the strengths of the columns having structural and intermediate grades of reinforcement are close together, while the line representing the strength for the columns with rail grade reinforcement lies considerably higher than the lines for the other grades.

In Fig. 33 the strengths of the columns have been plotted as ordinates and the total yield-point strength of the longitudinal steel as abscissas. Straight lines are obtained for each strength of concrete. The increase in

yield-point strength was due in some cases to change in grade of steel, and in other cases to increased percentage of reinforcement. Since for a given strength of concrete, all points group themselves along the same line, the strength of the column seemed to be independent of whether the change in yield point strength was affected by varying the percentage of a given grade of reinforcement ~~or~~ ^{or} by using different grades of reinforcement. The lines for different strength of concrete are practically parallel, indicating that the increase in strength due to the higher yield-point strength of the reinforcement was independent of the strength of the concrete. The slope of these lines represents the strength added to the columns for each unit of increase in strength of the reinforcement. The slope is, therefore, a measure of the effectiveness of the longitudinal reinforcement in adding strength to a column of a given concrete strength. The average slope of the curves in Fig. 33 indicate that the reinforcement added 97 percent of its yield-point strength to the strength of the concrete of the core. The yield-point stress of the steel was determined by the "drop of beam", a method which tends to give values slightly too high. The effectiveness of the longitudinal reinforcement may therefore be more than 97 percent of its yield-point strength.

In the further discussion in this report the longitudinal reinforcement has been assumed to add its full yield-point strength to the strength of the column. That is, it is considered 100 percent effective.

22. Effect of Rate of Loading on Deformation and

Strength of Columns - In Fig. 31 it is shown that the reinforced columns continued to deform during the 4-hour loading intervals. Up to the first load which was maintained for four hours, the deformations of the two sets of columns (designed for "fast" and "slow" loading) were practically equal in all cases, indicating good uniformity in the two sets of columns. The deformation during the first 4-hour interval was generally quite large. The deformation during the succeeding 4-hour intervals was not much different from the first one except near the maximum load, when the deformations became several times as large as during the previous intervals. During the application of an increment of load, the rate of deformation was so much lower than that for fast loading that the stress-strain curve was brought very nearly back to the curve for the fast loading.

Fig. 35, 36, and 37 show the deformation for fast and slow loading on columns reinforced with structural, intermediate, and rail grade longitudinal reinforcement. The grade

of reinforcement had no effect upon the deformation up to a load near the yield-point strength of the steel. The agreement between deformations at low loads for the two methods of testing is evident. In Fig. 37, showing the deformation of the columns having rail steel, the agreement between deformations found for fast and slow loadings was not so good as in the other cases.

The strengths of the columns tested with fast loading and slow loading are given in Tables 7 to 10. Table 7 gives the results for plain columns, and Tables 8, 9, and 10 for reinforced columns. The plain columns tested with slow loading generally developed less strength than did the companion columns with fast loading. The difference, however, was not very marked. For the reinforced columns the strength obtained with slow loading was very nearly equal to, but generally larger, than the strength obtained with fast loading.

23. Division of Load Between Concrete and

Reinforcement - Fig. 38 is designed to show the relation between the strengths of the reinforced columns and the sum of the strengths of the concrete and the reinforcement used. All the reinforced columns used in this figure had 4 percent longitudinal and 1.2 percent ^{spiral} reinforcement.

The sectional area of the concrete within the outer circumference of the spiral, that is, the core area minus the sectional area of the longitudinal bars was 48.3 sq. in., and the sectional area of the plain columns was ~~50.3~~^{53.5} sq.in. The plotted values in the lower curve are ~~50.3~~^{48.3} ~~53.5~~

times the strength of the plain columns, and represent the strength of the concrete in the reinforced columns. To these strengths have been added the total yield-point strengths of the longitudinal reinforcement. The strengths, ^{this obtained} for columns reinforced with structural, intermediate, and rail grades of reinforcement are shown as three distinct dotted lines. In addition, the strengths of the columns so reinforced but having in addition 1.2 percent spiral reinforcement are shown as solid lines. Based upon the assumption that the longitudinal reinforcement added its full yield-point strength to the column (as indicated in section 21), the vertical distance between the solid and the dotted lines for each grade of steel represents the strength added by 1.2 percent of spiral reinforcement. The solid line and the dotted line are not far from parallel, indicating that nearly a constant strength was added to the column by the spiral regardless of the strength of the concrete. The average total added strengths so determined were 16,700, 19,000, and 18,000 lb. for longitudinal reinforcement of structural, intermediate, and rail grades of steel respectively. Apparently the strength added by the

spiral was not affected by variations in the yield-point strength of the longitudinal reinforcement. The average of the added strengths for the three grades was 17,200 lb. for 1/2 percent of spiral, or 14,300 lb. for 1 percent. This is 70 percent of the average strength added by 1 percent of longitudinal reinforcement of intermediate grade as shown in Fig. 31.

The indication of the foregoing paragraphs is that the strength of the columns having longitudinal and spiral reinforcement was made up of (1) 85 percent of the cylinder strength times the net area of the concrete within the outer circumference of the spiral, (2) the yield-point stress of the longitudinal bars times their area, and (3) the strength added by the spiral, which amounted to about 70 percent of the strength added by an equal percentage of longitudinal reinforcement of intermediate grade steel.

24. Summary of Results - With the water content of the concrete constant at 39 gal. per cu.yd. the strength of the concrete increased in direct proportion to the increase in the cement content.

For all except the vibrated concrete, the increase in the modulus of elasticity was very nearly proportional to the increase in the compressive strength.

The strains measured on the steel agreed very well with those measured on the concrete for all columns, and the agreement between strains measured on columns of the same kind was good.

The average strength of the plain columns was approximately 85 percent of the strength of the cylinders and the uniformity of both the cylinders and the columns was good.

The stress-strain curves for the plain columns practically coincided with those for the corresponding control cylinders up to about one-half the maximum load. For higher loads the strains in the columns were somewhat greater than those in the cylinders.

For the 8000-lb. concrete the load ^{at} ~~for~~ any strain for columns with different percentages of longitudinal reinforcement was equal to the load carried by a column with no longitudinal reinforcement plus the load carried by longitudinal reinforcement having a modulus of elasticity of 50,000,000 lb. per sq.in. With the 3500-lb. concrete the conditions were the same as for the 8000-lb. concrete except that for any given strain the loads for the columns with all the different percentages of reinforcement were

a constant amount less than the sum of the load for the column without longitudinal reinforcement and the load found to be carried by the reinforcement.

For any two columns reinforced in the same manner, the difference in total strength was approximately equal to the difference in strengths of the concrete in the core as determined by the tests of the plain columns.

The increase in total strength due to the use of longitudinal reinforcement was approximately equal to the yield-point stress of the reinforcement times its sectional area, regardless of the grade of reinforcement used.

The strength of the plain columns tested with slow loading was generally slightly less than that of the columns tested with fast loading. For the reinforced columns the strength with slow loading was nearly equal to, but generally slightly greater than that for fast loading.

The indication of the tests is that the strength of the columns having longitudinal and spiral reinforcement was made up of (1) 85 percent of the cylinder strength times the net area of the concrete within the outer circumference of the spiral, (2) the yield-point stress of the longitudinal bars times their area, and (3) the strength added by the spiral, which amounted to about 70 percent of the strength added by an equal amount of longitudinal reinforcement of intermediate grade steel.

Table 1 - Properties of Concrete

Group No.	End Condition	Average Slump In.	28-Day Strength of Control Cylinders lb. per sq.in.								Average for Group
			Cylinders for Column A				Cylinders for Column B				
			1	2	3	Av.	1	2	3	Av.	
1	Plain	4-3/4	3075	2920	2770	2920	3510	3340	3200	3350	3140
2	Capital	4-1/4	3220	3030	3260	3170	3380	3740	3140	3420	3300
3	(20-dia.) (splice)	4-1/2	3460	3560	3500	3510	3460	3620	3440	3510	3510
4	(30-dia.) (splice)	3-1/2	3260	*	2980	3120	3340	3410	3280	3340	3230

* Broken Cap

* Broken Cap

✓

their

Table 2 - Strength of Columns and Control Cylinders

Group No.	Maximum Load, lb.			Strength of Control Cylinders lb. per sq. in.		
	Col. A	Col. B	Av.	Col. A	Col. B	Av.
1	252,800	265,000	258,900	2,920	3,350	3,140
2	260,000	251,000	255,500	3,170	3,420	3,300
3	218,600	245,400	232,000	3,510	3,510	3,510
4	231,000	231,000	231,000	3,120	3,340	3,230

Table 4 - Column Strength Corrected for Variation
in Concrete Strength

Group No.	Load Attributed to		Total Load	
	Concrete lb.	Steel lb.	lb.	percent
1	134,500	138,400	272,900	100
2	134,500	128,700	262,200	96.1
3	134,500	97,000	231,500	84.8
4	134,500	107,000	241,500	88.5

Division
 Table 3 - ~~Distribution~~ of Load Between Concrete and Reinforcement

Group No.	End Condition	Maximum Load		Load Attributed to Concrete		Load Attributed to Steel	
		lb.	percent	lb.	percent	lb.	percent
1	Plain	258,900	100	120,500		138,400	100
2	Capital	255,500	98.5	126,800		128,700	93.0
3	(20-dia.) (splice)	232,000	89.5	135,000		97,000	70.0
4	(30-dia.) (splice)	231,000	89.2	124,000		107,000	77.4

5
Table ① - Program of Tests for Series 2

Concrete Strength lb. per sq.in.			Reinforcement					Method of Loading	Number of Like Columns	Total No. of Columns		
			Longitudinal			Spiral						
			Grade	Per cent		Grade	Percent					
2000	3500	5000	-	0	4	-	0	Fast Slow	3	18		
2000	3500	5000	(Inter*) (Struc.) (Rail)		4	Inter.	1	Fast Slow	2	36		
	3500		Inter.	0	1.5	6	Inter.	1	Fast Slow	2	12	
	8000		-	0		-	0	Fast	3	3		
	8000		Inter.	0	1.5	4	6	Inter.	1	Fast	2	8
											Total 77	

Table ② - Summary of Tests of Reinforcement

Longitudinal Reinforcement

Nominal Size	No. of Coupons Tested	Yield Point Stress lb. per sq. in.	Ultimate Strength lb. per sq. in.
--------------	-----------------------------	---------------------------------------	--------------------------------------

Intermediate Grade

1/2-in. square	28	43,560	66,960
1/2-in. <i>round</i> diam.	6	51,640	76,550
5/8-in. <i>round</i> diam.	6	45,540	73,660
3/4-in. <i>round</i> diam.	6	45,470	69,220

Structural Grade

1/2-in. square	24	40,840	64,320
----------------	----	--------	--------

Rail *Grade*
Steel

1/2-in. square	21	65,320	102,680
----------------	----	--------	---------

Spiral Reinforcement

Intermediate Grade

No. 5 (0.207 diam.)	65	Not Determined	82,760
---------------------	----	----------------	--------

7
Table ④ - Strength of Plain Concrete Columns
Under Fast Loading and Slow Loading.

Strengths are given in pounds per square inch.

Method of Loading Columns	Strength of Control Cylinders - lb. per sq. in.				
	For Column No.			Average	Designed Strength
	1	2	3		
Fast	2120	2000	2210	2110	2000
Slow	2250	2220	2080	2180	2000
Fast	3000	3600	3420	3340	3500
Slow	3220	3450	3450	3370	3500
Fast	5050	5260	5280	5200	5000
Slow	5130	5120	5480	5240	5000
Fast	7470	7450	7060	7330	8000

Method of Loading Columns	Strength of Columns				Per cent Cylinder Strength
Fast	1820	1850	1680	1780	84.5
Slow	1680	1680	1680	1680	77.8
Fast	2790	3010	3050	2950	88.5
Slow	2800	2520	2800	2710	80.5
Fast	4290	4470	4130	4300	82.8
Slow	4770*	4770*	----**	4770*	91.1*
Fast	6680	5930	7070	6560	89.5
				Av.	85.0

* Accidentally run beyond the last 10% load increment

** Tipped over and broken before the testing

Table ③ - Effect of Grade of Longitudinal Reinforcement
on Strength of Columns

Reinforcement in all columns: 4% Longitudinal and ¹²1% Spiral

Designed	Concrete Strength			Load on Column at Failure			Method of Loading
	Obtained			1	2	Av.	
	1	2	Av.				
2000	<u>Intermediate Grade</u>						
2000	2360	2340	2350	221,000 × 1.5 hr.	214,200 3.5 hr.	217,600 2.5 hr.	Fast
2000	2360	2400	2380	210,000	210,000	210,000	Slow
3500	3790	3660	3730	260,500 × 15 min.	268,000 15 min.	264,250 15 min.	Fast
3500	3810	3980	3900	275,000	275,000	275,000	Slow
5000	5150	5410	5280	321,000 × 0	297,500 5 min.	309,250	Fast
5000	5160	5670	5420	330,000	300,000	-	Slow
	<u>Structural Grade</u>						
2000	2250	2360	2310	201,500 × 5 min.	202,700 40 min.	202,100 25 min.	Fast
2000	2320	2370	2350	210,000	210,000	210,000	Slow
3500	3750	3810	3780	252,600 × 25 min.	253,000 30 min.	252,800	Fast
3500	3830	3800	3820	225,000	250,000	-	Slow
5000	5180	5350	5270	311,500 × 0	308,000 45 min.	309,750	Fast
5000	5270	5480	5380	330,000	300,000	-	Slow
	<u>Rail Steel</u>						
2000	2380	2230	2310	239,300 max.	244,700 × 7 min.	242,000	Fast
2000	2260	2520	2390	260,200	255,000	-	Slow
3500	3650	3740	3700	307,000 × 25 min.	304,700 0	305,850	Fast
3500	3800	3780	3790	300,000	325,000	-	Slow
5000	5460	5620	5540	355,000 × 0	378,000 0	366,750 0 min.	Fast
5000	5530	5800	5670	390,000	390,000	390,000	Slow

* Time from application of maximum Load until failure

2-5

Table 5 - Effect of Variation in Percentage
of Longitudinal Reinforcement
on Strength of Columns

Fast Loading

<u>Percent Reinforcement</u> Long. Spiral	<u>Cylinder Strength</u> lb. per sq. in.	<u>Maximum Load at Failure</u> lb.	<u>Assumed Load Distribution</u> Concrete 85% of Strength lb.	<u>Distribution Attributed to Presence of Reinforcement</u> lb.	<u>Nominal Area of Long. Reinf.</u> sq. in.
--	---	---------------------------------------	---	--	--

Designed Cylinder Strength, 8000 lb. per sq. in.

0	0	7330	348,300	348,300	0	0
0	1.2	6590	350,400	282,000	68,400	0
1-1/2	1.2	6600	400,150	278,000	122,150	0.785
4	1.2	7070	421,500	290,000	131,500	2.00
6	1.2	6950	470,000	280,000	190,000	2.99

Designed Cylinder Strength, 3500 lb. per sq. in.

0	0	3340	157,900	157,900	0	0
0	1.2	3760	185,700	160,700	25,000	0
1-1/2	1.2	3410	213,900	143,500	70,400	0.785
4	1.2	3730	264,250	153,000	111,250	2.00
6	1.2	3770	308,450	151,450	157,000	2.99

Table 10 - Effect of Variation in Percentage of Longitudinal Reinforcement
on Strength of Columns

Designed strength of concrete, 3500 lb. per sq.in.

Spiral
Longitudinal reinforcement, 1.2 percent

Percent Longitudinal Reinforcement	Concrete Strength Obtained lb. per sq.in.			Load on Columns lb.			Method of Loading
	1	2	Av.	1	2	Av.	
0.0	3790	3730	3760	186,900	184,500	185,700	Fast
0.0	3920	3810	3870	15 min. 200,000	85 min. 180,000	---	Slow
1.5	3710	3100	3410	218,500	209,300	213,900	Fast
1.5	3920	3800	3860	15 min. 220,000	0 240,000	---	Slow
4.0	3790	3660	3730	260,500	268,000	264,250	Fast
4.0	3810	3980	3900	15 min. 275,000	15 min. 275,000	15 min. 275,000	Slow
6.0	3850	3680	3770	305,000	311,900	308,450	Fast
6.0	3690	3790	3740	15 min. 300,000	5 min. 300,000	10 min. 300,000	Slow

*omit all
parentheses
see*